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Numerical study on progressive failure of hard rock samples with an unfilled undulate joint

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ABSTRACT

Roughness is one of the most important parameters of unfilled joint. In this study, a series of numerical tests have been carried out on the progressive failure of rock masses with varied undulate joints. The numerical model reproduced the stress–strain relations obtained in lab tests on the marble samples with smooth joint under the confinement of $\sigma_3/\sigma_c = 0.013$ and 0.27; meanwhile, the results of the numerical model on rock mass with rough joint indicated that with the increase of confining stress, the failure of rock mass presents as sliding along the rock joint (Type I), shearing partly through asperity and the joint (Type II) and mostly shearing through the rock block initiated by joint (Type III) respectively. To describe the progressive failure of hard rock samples with an unfilled undulate joint, an index of stress concentration factor (SCF) is presented as the ratio of σ_1–σ_3 (the localized principal stress difference of one point in the rock sample) to $(\sigma_1$–$\sigma_3)_o$ (the principal stress difference applied onto the rock sample) to denote the degree of heterogeneous stress induced by the undulate joint. It has been found that the progressive failure behavior is largely dependent on the joint roughness and the confining stress, which can be classified into 4 modes as: slipping plastic failure, slipping–shearing brittle failure, shearing brittle failure and shearing plastic failure. For the rock mass with rough joint, the crack initiation and propagation resulted from localized stress concentration at the turning point of rough joint during compression plays an important role in the progressive failure. For the rock mass with the same rough joint, SCF_m decreases as the confining pressure increases because the confinement can weaken the heterogeneity of the rock mass.

1. Introduction

Attention was not focused on the discontinuities in natural rock mass until some large disasters induced by failure along weak planes occurred, e.g., Malpasset Dam accident (1959) and Vajont Dam accident (1963). After over 50 years, it is a common view that discontinuities (joints, bedding planes, faults et al.) that run through the natural rock mass always dominate the mechanical behaviors of rock mass (Jaeger, 1960; Müller, 1974; Sun, 1988).

Generally, rock joints can be divided into two types — filled and unfilled joints. The mechanical behaviors of filled joints are strongly influenced by the characteristics of infillings, such as cohesion, friction and thickness of materials between joint walls. On the contrary, the mechanical behaviors of unfilled joints are largely dependent on the effective normal stress and the properties of joint walls, such as rock type, degree of roughness, the size of joint and weathering degree of joint walls.

The unfilled joints are widely observed in the natural slopes, tunnels or other rock engineering fields. The joints are always not smooth but undulate and inter-locked under normal stress. It is very important in rock engineering practice to make it clear that how the roughness of rock joint influences on rock mass behaviors.

There are mainly four models to describe the strength of a joint: Coulomb model, Patton model, Barton model and Byerlee model. The Coulomb model was proposed firstly in the eighteenth century, which indicates that the relationship between shear and normal stress of two smooth surfaces. Patton (1966) firstly considered the contribution of the roughness on the strength of the joints, and presented a bilinear strength envelop to describe the shear strength. In Patton model (1966), at a low normal stress σ_n less than σ_T, the shear strength can be expressed as:

$$\tau = \sigma_n \tan\ (\varphi_b + i) \tag{1}$$

where φ_b is the basic friction angle; i is the inclination angle of the teeth (Fig. 1), while at a high normal stress σ_n greater than σ_T, the shear stress can be expressed as Eq. (2) as a result most asperities were sheared off.

$$\tau = c_j + \sigma_n\ \tan\ \varphi_r \tag{2}$$

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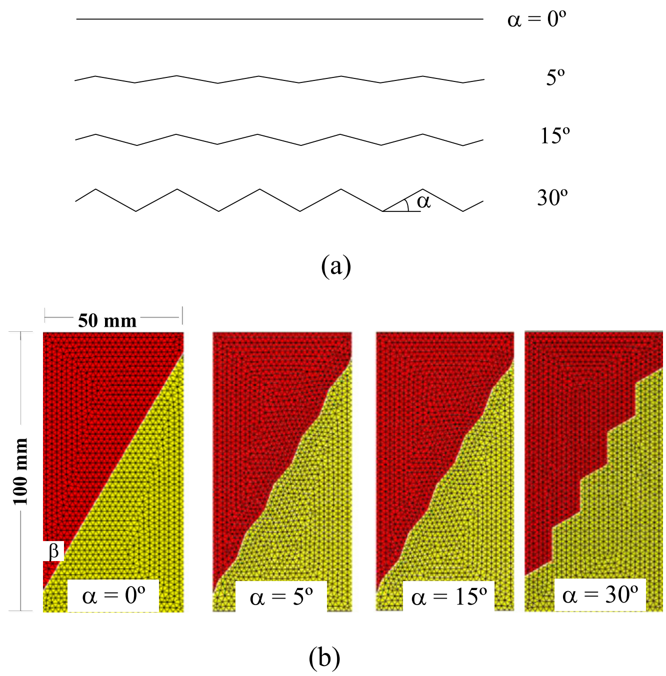

Fig. 1. Geometry of rock-mass models. (a) Smooth and varied saw-cutting joint models; (b) The rock mass models with varied rough joints in FLAC^{3D}.

where c_j is the apparent joint cohesion and φ_r is the residual friction angle, and

$$\sigma_T = \frac{c_j}{\tan(\varphi_b + i) - \tan\varphi_r}.$$

Following Patton (1966), Ladanyi and Archambault (1969) and Saeb (1990) considered the influence of the different parts of sliding and breaking of asperities and put forward a modified criterion, Haberfield and Johnston (1994) considered irregularity of joint profile and tried to predict the variation of dilatancy angle with shear displacement, and Maksimovic (1992) developed Patton's model to natural profiles.

Barton (1976) proposed two empirical equations (Eqs. (3) and (4)) to estimate the strength of a joint at low normal stress, i.e. $0.01 < \sigma_n/JCS < 0.3$ and at high normal stress i.e. $\sigma_n \geq \sigma_c$ respectively,

$$\tau = \sigma_n \tan\left[JRC \ \log\left(\frac{JCS}{\sigma_n}\right) + \varphi_b\right] \tag{3}$$

$$\tau = \sigma_n \tan\left[JRC \ \log\left(\frac{\sigma_1 - \sigma_3}{\sigma_n}\right) + \varphi_b\right] \tag{4}$$

where σ_c is the uniaxial compressive strength, JRC is joint roughness coefficient, JCS is joint wall compressive strength, φ_b is basic friction angle, and σ_1 and σ_3 are major and minor principle effective stresses, respectively. Zhao (1997a, 1997b) modified JRC models for the mismatched joints.

Based on a great deal of experimental data, Byerlee (1978) found that the friction of rock surface under low normal stress is strongly dependent on surface roughness, and JRC model of Eq. (3) proposed by Barton (1976) and Barton and Choubey (1977) can be used for such low stress state, e.g. $\sigma_n \leq 5$ MPa; under higher normal stress, the friction can be estimated by the following empirical formulae as:

$$\tau = 0.85\sigma_n, \quad 5 \text{ MPa} \leq \sigma_n \leq 200 \text{ MPa} \tag{5}$$

$$\tau = 0.5 + 0.6\sigma_n, \quad 200 \text{ MPa} \leq \sigma_n \leq 2000 \text{ MPa}. \tag{6}$$

Eqs. (5) and (6) indicate that the effect of rock joints is weakened and the joint roughness has little or no effect on friction under high normal stress. This effect is also confirmed by lab tests from Brown (1970), Zhou (1985) and Ramamurthy and Arora (1994).

Besides, some other models have been proposed by researchers, such as the negative-exponential model proposed by Grasselli and Egger (2003).

It can be seen that much attention has been paid on the joint roughness since the 1960s (Patton, 1966; Barton and Choubey, 1977), mostly focusing on the strength and dilation deformation during shear failure at low normal stress. However, the mechanical behavior of rock mass with rough joint at high confining stress is still not very clear and few studies were conducted on progressive failure of undulate joint. As the overall axial stress increases, it is still not clear how the localized stress gets concentrated and transferred, which leads to crack initiation, propagation and finally failure.

In this study, a series of numerical triaxial compression tests were conducted to study the progressive failure of rock mass with a single inclined smooth or undulate joint. In this condition the numerical simulation can be affected by lots of factors such as specimen length, the location and orientation of the inclined joint, the characteristics of the undulant joint, the boundary constraints, and even the persistent joint (Xu et al. 2013; Wasantha et al., 2014). This study focused on the roughness of joints and fixed the other factors to try to understand the progressive failure of rock mass with various undulate joints under different confining stresses.

2. Rock models with undulate joints

2.1. Models of rock specimens

Rock mass with various undulate joints would be considered in the study. Our study has been carried out with a mature numerical method — FLAC^{3D} (Itasca Consulting Group Inc, 2005), and a smooth joint is represented with interface element. The properties of the intact rock and the joint can be inputted directly to the corresponding segments in the numerical models.

It is known that the scale effect of joint asperities plays an important role in the results of numerical simulations. To overcome the influence of the scale effect, the number of asperities is set as identical in each model to focus on the roughness of joints, i.e. the inclination angles in this study. Four joints with smooth and varied saw-cutting shapes are built as shown in Fig. 1a. It can be seen that the inclination angles of the joints are set as 0° (smooth), 5°, 15° and 30°, respectively. The angle between joint and axial loads is set as 30°. The trace length of the joints is 10 cm and the size of the rock sample is ϕ 5 cm × 10 cm. The rock mass models with a single rough joint in FLAC^{3D} are shown in Fig. 1b. There are about 3500 elements in each model. In FLAC^{3D}, the saw-cutting shaped joint is treated as several linked smooth segments, and each segment is modeled with a corresponding interface element. During tests, a servo control is used with the applied strain rate (i.e. about 10^{-6}) to restrict the maximum unbalanced force. For the triaxial tests, the confining pressure is applied on a stiff slate around the rock sample to keep the pressure uniform.

2.2. Intrinsic properties of rock mass

The parameters of coarse marble from Jinping Hydropower Station are adopted as that of rock blocks in the numerical simulations. Guo et al. (2013b) found that cohesion of the coarse marble is lost while friction is mobilized with the increase of the plastic strain by cycling tests under varied confining pressures, and the strength can be described with a model of cohesion weakening and friction strengthening (CWFS model) (Martin, 1997; Hajiabdolmajid et al., 2002). In this numerical study, Mohr–Coulomb model is applied considering the process of cohesion weakening and friction mobilizing with the increase of